VOLUME 81 SEPARATE No. 683

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

MAY, 1955

BENDING IN ANNULAR SECTIONS

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STRUCTURAL DIVISION

{Discussion open until September 1, 1955}

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Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

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This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

BENDING IN ANNULAR SECTIONS

Adolph A. Marrone,[1] M. ASCE

SYNOPSIS

The distribution of stresses on an annular section under an eccentric load is derived by making use of the properties of circular arcs, on the assumption that the thickness of the annulus is small compared to the radius. The resulting formulas are extended to reinforced concrete sections subjected to bending only, or to loads with any degree of eccentricity.

Charts for convenience in design are presented, together with correction curves for thicker sections.

INTRODUCTION

The problem of bending in an annular section arises in the analysis of foundations having the shape of a hollow circle and subjected to an eccentric load, masonry or reinforced concrete chimneys under wind or earthquake loads, and long reinforced concrete pipe with lateral loads as in a beam.

In the development of formulas for determining the stresses on such sections the assumption is made that the thickness of the wall, t, is small compared to the radius, R, to the center of the wall so that formulas for the properties of a circular arc may be applied.

Symbols and Equations

The meaning of the symbols used in this paper either is explained in the text where they first occur, or is indicated in the figures.

The equations have been left in the form in which it is believed they are most adapted to step by step computations for construction of the various charts.

Case I Compression over the Whole Section $0 \leq \frac{E}{R} \leq \frac{1}{2}$

The general formula for stress f in a section subject to axial load N and to bending M, when compression is present over the whole section, is

(1) $$f = \frac{N}{A} \pm \frac{NEc}{I}$$ where

(2) $$E = \frac{M}{N}$$ is the eccentricity

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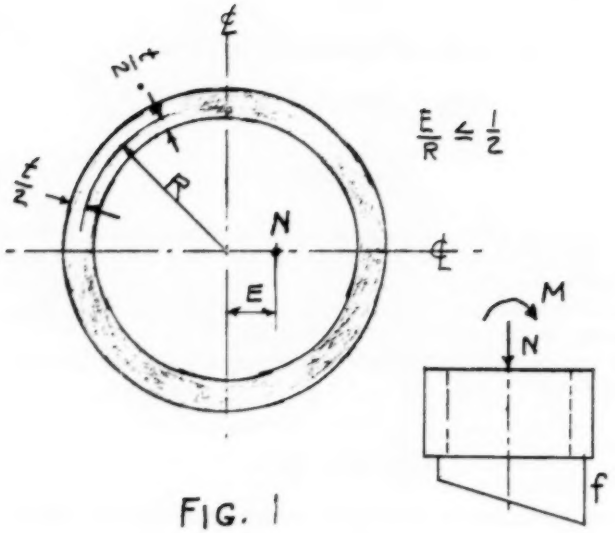


FIG. 1

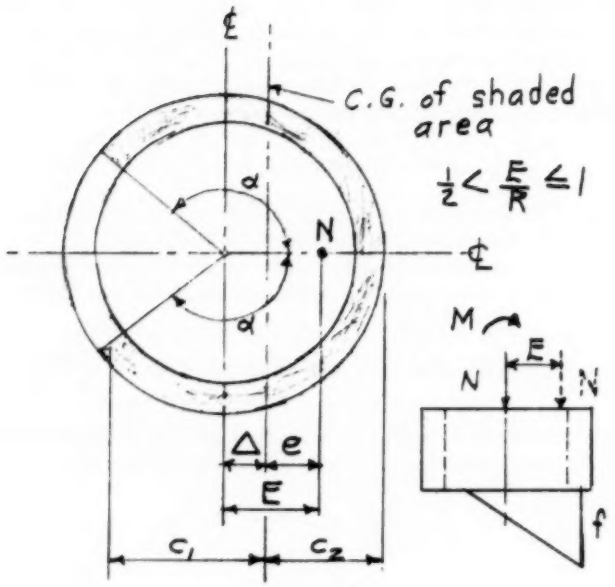


FIG. 2

of the load and c is the distance to the point at which the stress is being determined, both measured from the center of gravity axis. For compression over the whole section the properties of the whole section apply as in Fig. 1 so that

$$A = 2\pi Rt \qquad I = \pi R^3 t \qquad c_1 = c_2 = R$$ under

the assumption that t is small compared to R. Thus for Case I the formula for the maximum and minimum compressive stresses is

(3) $$f = \frac{N}{2\pi Rt} \pm \frac{NER}{\pi R^3 t} = \frac{N}{\pi Rt}\left(\frac{1}{2} \pm \frac{E}{R}\right)$$

The limit of application for Case I and Eq. (3) is when the stress on the windward side is zero. Therefore for Case I, compression over the whole section

(4) $$0 \leq \frac{E}{R} \leq \frac{1}{2}$$

Case II The Material is Capable of Resisting Compression Only. Annular shaped foundations and masonry chimneys come under this Case.

For this Case a portion of the section on the windward side is unstressed; the general formula of Eq. (1) is still applicable provided the properties of the reduced section are used.

(5) $f = \frac{N}{A} \pm \frac{Nec}{I}$ where $e = E - \Delta$. In this case the stress on the windward side at a distance c_1 from the center of gravity, Fig. 2, is zero.

$$0 = \frac{N}{A} - \frac{Nec_1}{I}$$ Solving for e

$$\frac{I}{Ac_1} = e = E - \Delta$$ and therefore

(6) $$E = \frac{I}{Ac_1} + \Delta$$

Substituting Eq. (6) in Eq. (5) to get the maximum stress on the leeward side:

$$f = \frac{N}{A} + \frac{N}{I}(E - \Delta)c_2$$

(7) $$f = \frac{N}{A}\left(1 + \frac{c_2}{c_1}\right)$$

In Eq. (7) A the area of the stressed section and the ratio $\frac{c_2}{c_1}$ are not known since they depend on the angle α (See Fig. 2). Furthermore in Eq. (6) $E = \frac{M}{N}$ is known, but all the terms on the right hand side of the equation are unknown and depend on α. If values of I, A, c_1, and Δ are computed for various values of α, $E = \frac{I}{Ac_1} + \Delta$ can be computed and tabulated against α. Then knowing E for a particular problem we can find α compute $A = Z\alpha Rt$ and the ratio $\frac{c_2}{c_1}$ and determine f in Eq. (7).

The formulas for the properties of a sector of a thin annulus subtending on angle 2α as shown in Fig. (2) are given by Roark "Formulas For Stress and Strain," McGraw-Hill 1943 page 70:

(8) $$I = R^3 t\left(\alpha + \sin\alpha\cos\alpha - \frac{2\sin^2\alpha}{\alpha}\right)$$

(9) $$c_1 = R\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right)$$

(10) $$c_2 = R\left(1 - \frac{\sin\alpha}{\alpha}\right)$$

(11) $$\Delta = R\frac{\sin\alpha}{\alpha}$$

(12) $$A = 2\alpha Rt$$

Values of I, c_1, c_2, Δ and A computed from Eqs. (8), (9), (10), (11) and (12) respectively for various values of the angle α are shown in Table I in dimensionless form. Also shown are values of E computed from Eq. (6). Eq. (7) may be written

$$\frac{N}{fRt} = \frac{2\alpha\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right)}{1 - \cos\alpha}$$

(13) $$\frac{N}{fRt} = \frac{2\alpha c_1}{c_1 + c_2}$$

and its values are also shown in Table 1.

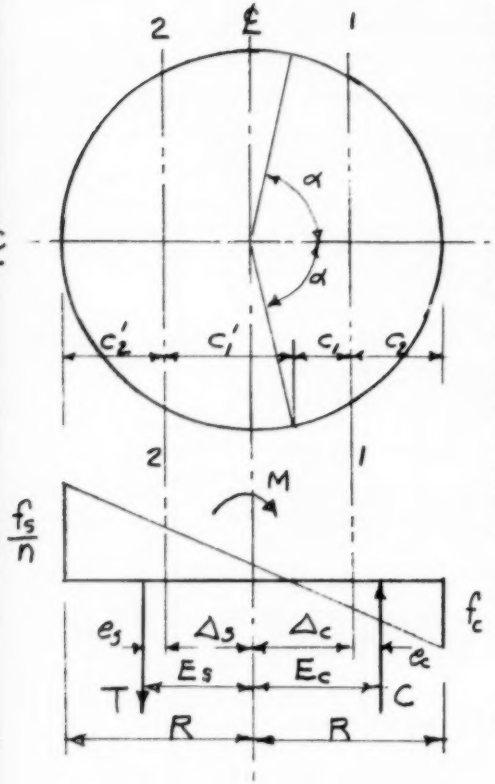
The variation of $\frac{E}{R}$ with α is shown in Chart 1 and the variation of the maximum stress f with $\frac{E}{R}$ is shown in Chart 2.

α Deg.	α Rad.	$\frac{I}{R^3t}$	$\frac{\bar{y}}{R}$	$\frac{C_1}{R}$	$\frac{C_2}{R}$	$\frac{A}{Rt}$	$\frac{\bar{x}}{R}$	$\frac{N}{fRt}$
0	0	0	1	0	0	0	1	0
5	.08727	3×10^{-7}	.99873	.00254	.00127	.1745	.99940	.1165
10	.1745	72×10^{-7}	.99493	.01012	.00507	.3491	.99697	.2325
15	.2618	594×10^{-7}	.98862	.02269	.01138	.5236	.99362	.3487
20	.3491	227×10^{-6}	.97982	.04013	.02018	.6981	.98792	.4646
25	.4363	683×10^{-6}	.9686	.06226	.03143	.8727	.98114	.5799
30	.5236	1683×10^{-6}	.9549	.08890	.04507	1.0472	.97300	.6949
35	.6109	.00358	.9390	.1198	.06104	1.2217	.9634	.8094
40	.6981	.00687	.9207	.1547	.07927	1.3963	.9525	.9232
45	.7854	.01216	.9003	.1932	.09968	1.5708	.9404	1.0362
60	1.0472	.04782	.8270	.3270	.1730	2.0944	.8968	1.3697
75	1.3090	.13346	.7379	.4791	.2621	2.6180	.8443	1.6922
90	1.5708	.29756	.6366	.6366	.3634	π	.7854	2.0000
105	1.8326	.56434	.5271	.7859	.4729	3.6652	.7230	2.2882
120	2.0944	.94517	.4135	.9135	.5865	4.1888	.6605	2.5510
135	2.3562	1.4318	.3001	1.0072	.6999	4.7124	.6018	2.7803
140	2.4435	1.6129	.2631	1.0291	.7369	4.8869	.5838	2.8477
145	2.5307	1.8009	.2267	1.0458	.7733	5.0614	.5668	2.9097
150	2.6180	1.9940	.1910	1.0570	.8090	5.2360	.5513	2.9659
155	2.7053	2.1902	.1562	1.0625	.8438	5.4105	.5372	3.0157
160	2.7925	2.3874	.1225	1.0622	.8775	5.5851	.5249	3.0584
165	2.8798	2.5833	.08987	1.0558	.9101	5.7596	.5147	3.0932
170	2.9671	2.7728	.05853	1.0433	.9415	5.9341	.5064	3.1193
175	3.0543	2.9625	.02854	1.0247	.9715	6.1087	.5018	3.1358
180	π	π	0	1	1	2π	.5000	π

TABLE 1. PROPERTIES OF A CIRCULAR ARC

Case III Reinforced Concrete in Bending Only

The theory and properties of sectors of thin annular sections developed in Case II can be applied to determine the stresses in the concrete and the longitudinal steel of structures such as reinforced concrete pipes subject to bending.



It will be assumed that the thickness t is small compared to the radius R of the section, that the cross sectional area of steel per inch of perimeter A_s is uniform over the tension area, and that tension is resisted by the steel only.

f_s and f_c are the maximum unit stresses in the steel and concrete respectively on the ℄ in the plane of the moment M. C is the resultant of the concrete stresses with an eccentricity e_c from the c.g. axis 1-1 of the compression area, T is the resultant of the steel stresses with an eccentricity e_s from the c.g. axis 2-2 of the tension area. The neutral axis is defined by the angle α, and c_1, c_1', c_2 and c_2' are distances as shown in Fig. 3.

Referring to Fig. 3 and applying Eq. (7):

$$(14)\qquad f_c = \frac{C}{2\alpha Rt}\left(1+\frac{c_2}{c_1}\right)$$

and $$f_s = \frac{T}{2(\pi-\alpha)RA_s}\left(1+\frac{c_2'}{c_1'}\right)$$

For equilibrium

$$M = C(E_s+E_c) = T(E_s+E_c)$$

and substituting in Eq. (14) we get

$$(15)\qquad f_c = \frac{M}{2\alpha Rt(E_s+E_c)}\left(1+\frac{c_2}{c_1}\right) = \frac{M}{Rtk}$$

$$(16)\qquad f_s = \frac{M}{2(\pi-\alpha)RA_s(E_s+E_c)}\left(1+\frac{c_2'}{c_1'}\right) = \frac{M}{RA_s j} \quad \text{where}$$

(17) $$k = \frac{2\alpha}{1 + \frac{c_2}{c_1}}(E_s + E_c) \quad \text{and} \quad j = \frac{2(\pi - \alpha)}{1 + \frac{c_2'}{c_1'}}(E_s + E_c)$$

From similar triangles in the stress diagram and by dividing Eq. (15) by Eq. (16)

(18) $$\frac{f_c}{f_s/n} = \frac{R(1 - \cos\alpha)}{R(1 + \cos\alpha)} = n\frac{A_s}{t}\frac{\pi - \alpha}{\alpha}\frac{1 + \frac{c_2}{c_1}}{1 + \frac{c_2'}{c_1'}}$$

Denoting $\frac{A_s}{t}$ by p the ratio of steel area to concrete area

$$np = \frac{1 - \cos\alpha}{1 + \cos\alpha}\frac{\alpha}{1 + \frac{c_2}{c_1}}\frac{1 + \frac{c_2'}{c_1'}}{\pi - \alpha}$$

By applying Eqs. (9) and (10) this reduces to

(19) $$np = \frac{\alpha c_1}{(\pi - \alpha)c_1'} = \frac{\sin\alpha - \alpha\cos\alpha}{\sin\alpha + (\pi - \alpha)\cos\alpha}$$

Equation (19) determines the location of the neutral axis as defined by the angle α when only bending is present. For any value of α $E_c = E/\alpha$ and $E_s = E/\pi - \alpha$ are determined from Chart 1, and from Eqs. (9) and (10) c_1 and c_2 are computed for α and c_1' and c_2' are computed for $\pi - \alpha$.

These values are substituted in Eqs. (17) to compute k and j. This has been done for various values of α and $\frac{k}{R} = \frac{M}{f_c t R^2}$ and $\frac{j}{R} = \frac{M}{f_s A_s R^2}$ are plotted against np in Chart 3.

For brevity the above derivation was based on there being no steel in the compression zone. With steel uniformly distributed throughout the entire section, C in Eq. (14) would be the sum of the compression in the concrete C_c and the compression in the compression steel C_s.

$$C = C_c + C_s = \frac{(1-p)2\alpha R t f_c}{1 + \frac{c_2}{c_1}} + \frac{2\alpha R t p(n f_c)}{1 + \frac{c_2}{c_1}}$$

$$= \frac{2\alpha R t}{1 + \frac{c_2}{c_1}} f_c\left[1 + (n-1)p\right]$$

Solving for f_c and substituting in Eq. (18) we get for the expression for the neutral axis when compression steel is present

(20) $$\frac{np}{1 + (n-1)p} = \frac{\alpha c_1}{(\pi - \alpha)c_1'}$$

Thus Chart 3 serves also for sections with compression steel provided it is entered with the value $\frac{np}{1 + (n-1)p}$ instead of np.

For balanced steel ratio the value $\frac{f_c}{f_s/n}$ in Eq. (18) is fixed and is equal to $\frac{1 - \cos\alpha}{1 + \cos\alpha}$

From Eqs. (15) and (16)

$$\frac{f_c}{f_s} = \frac{A_s}{t}\,\frac{j}{k} = p\frac{j}{k}$$ from which

(21) $p\frac{f_s}{f_c} = \frac{k}{j}$ Thus for a given value of α, $n\frac{f_c}{f_s}$ is computed from $\frac{1-\cos\alpha}{1+\cos\alpha}$ and $P\frac{f_s}{f_c} = \frac{k}{j}$ is computed from the ratio of k to j as given by Eqs. (17). This has been done for various values of α and the results plotted in Chart 4.

Case IV Reinforced Concrete with Bending and Axial Load.

The same assumptions are made here as in Case III. Structures such as reinforced concrete chimneys are covered in this case. When $0 \leq \frac{E}{R} \leq \frac{1}{2}$ the whole section will be under compression and the structure falls in Case I. Case IV is concerned with the situation where $\frac{1}{2} \leq \frac{E}{R} < \infty$ Fig. 4 is the same as Fig. 3 except the axial load is added. Taking moments about T we have $N(E + E_S) - C(E_S + E_C) = 0$

$$C = \frac{N(E+E_s)}{E_s+E_c}$$

similarly by taking moments about C

$$T = \frac{N(E-E_c)}{E_s+E_c}$$

Substituting these expressions in Eq. (7)

(22) $$f_c = \frac{N(E+E_s)}{2\alpha Rt(E_s+E_c)}\left(1 + \frac{c_2}{c_1}\right)$$

(23) $$f_s = \frac{N(E-E_c)}{2(\pi-\alpha)RA_s(E_s+E_c)}\left(1 + \frac{c_2'}{c_1'}\right)$$

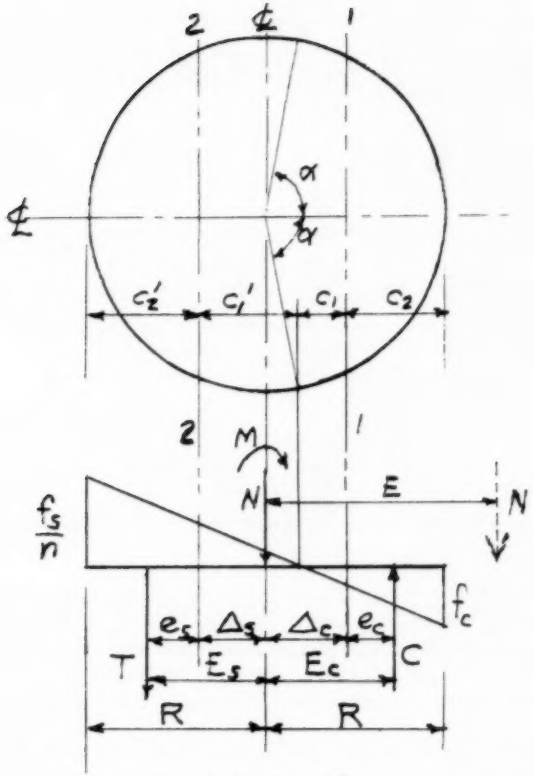

2
1
α
α
c_2'
c_1'
c_1
c_2
M
N
E
N
$\frac{f_s}{n}$
f_c
e_s
Δ_s
Δ_c
e_c
T
C
E_s
E_c
R
R

FIG. 4

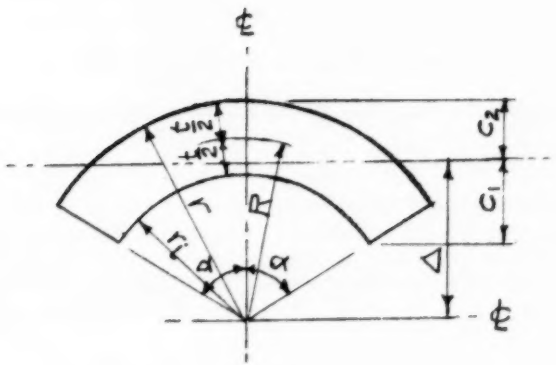

$\frac{\alpha}{2}$
$\frac{\alpha}{2}$
r
R
r_1
α
α
c_2
c_1
Δ

Fig. 5

From similar triangles and dividing Eqs. (21) and (22).

$$\frac{f_c}{f_s/n} = \frac{R(1-\cos\alpha)}{R(1+\cos\alpha)} = np\,\frac{\pi-\alpha}{\alpha}\,\frac{E+E_s}{E-E_c}\,\frac{1+\frac{C_2}{C_1}}{1+\frac{C_2'}{C_1'}}$$

where $p = \frac{A_s}{t}$. Solving for $\frac{E}{R}$

$$\frac{E}{R} = \frac{\frac{E_s}{R} + \frac{E_c}{R}\,\frac{\alpha}{np(\pi-\alpha)}\,\frac{1-\cos\alpha}{1+\cos\alpha}\,\frac{1+\frac{C_2'}{C_1'}}{1+\frac{C_2}{C_1}}}{\frac{\alpha}{np(\pi-\alpha)}\,\frac{1-\cos\alpha}{1+\cos\alpha}\,\frac{1+\frac{C_2'}{C_1'}}{1+\frac{C_2}{C_1}} - 1}$$

(24)
$$\frac{E}{R} = \frac{\frac{E_s}{R} + \frac{E_c}{R}\,\frac{np_\alpha}{np}}{\frac{np_\alpha}{np} - 1}$$

where np_α is given by Eq. (19). Eq. (23) determines the neutral axis; for a given value of np and α $E_S = E/\pi - \alpha$ and $E_C = E/\alpha$ are determined from Chart 1, and np_α is computed by Eq.(19). These values are then substituted in Eqs. (22) and (23) to obtain f_c and f_s. This was done for various values of α and np and the resulting stresses are plotted in Charts 5 and 6 against $\frac{E}{R}$.

For a complete treatment of reinforced concrete chimneys see the report of American Concrete Institute Committee 505 ("Proposed Standard Specification for the Design and Construction of Reinforced Concrete Chimneys," AC1 Proceedings for January 1953). The report also includes consideration of stresses due to temperature, and the case of chimneys with an opening in the side.

Error Caused by Treating Section as an Arc

Consider a sector of a thick annulus as shown in Fig. 5. Equation (7) gives the maximum stress as $f = \frac{N}{A}\left(1 + \frac{c_2}{c_1}\right)$ but c_1, c_2 and Δ must be for a thick annulus. It can be easily shown that

(25)
$$\Delta = \frac{2\sin\alpha}{3\alpha}\left(\frac{r^3 - r_i^3}{r^2 - r_i^2}\right)$$

(26)
$$c_1 = \frac{2\sin\alpha}{3\alpha}\left(\frac{r^3 - r_i^3}{r^2 - r_i^2}\right) - \begin{cases} r_i\cos\alpha & \text{for } \alpha \leq 90° \\ r\cos\alpha & \text{for } \alpha > 90° \end{cases}$$

(27)
$$c_2 = r - \frac{2\sin\alpha}{3\alpha}\left(\frac{r^3 - r_i^3}{r^2 - r_i^2}\right)$$

Also $R = \frac{r + r_i}{2}$ $\quad t = mR$ from which

(28)
$$r = \frac{R}{2}(2+m) \qquad r_i = \frac{R}{2}(2-m)$$

Substituting Eqs. (28) in Eq. (26) and Eq. (27) gives

(29)
$$c_1 = R\left[\frac{\sin\alpha}{\alpha}\left(1+\frac{m^2}{12}\right) - \begin{cases}\left(1-\frac{m}{2}\right)\cos\alpha & \text{for } \alpha \leq 90^\circ \\ \left(1+\frac{m}{2}\right)\cos\alpha & \text{for } \alpha > 90^\circ\end{cases}\right]$$

$$c_2 = R\left[1 + \frac{m}{2} - \frac{\sin\alpha}{\alpha}\left(1+\frac{m^2}{12}\right)\right]$$

Expressing the error as a fraction of the stress computed on the basis of a thin annulus

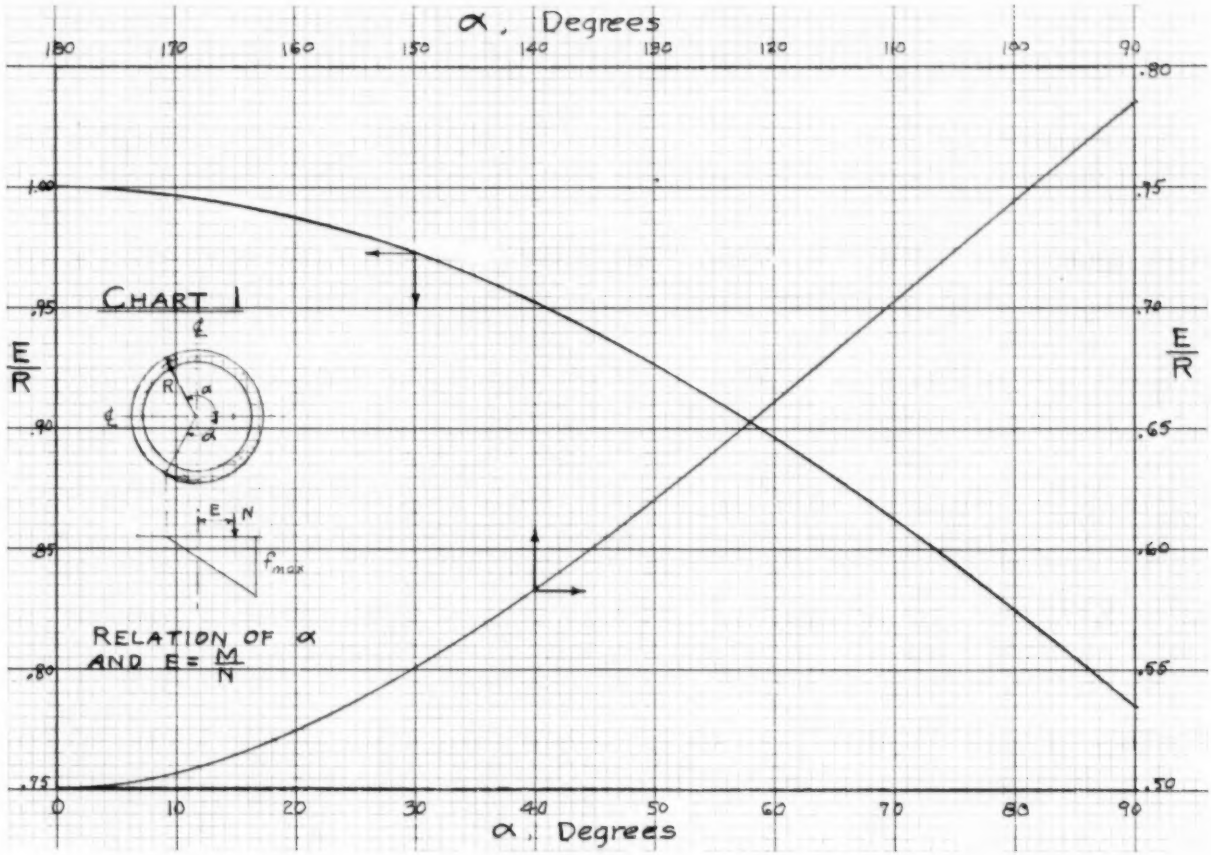
$$\text{Error} = \frac{f - f_a}{f_a} = \frac{\frac{N}{A}\left(1+\frac{c_2}{c_1}\right) - \frac{N}{A}\left(1+\frac{c_{2a}}{c_{1a}}\right)}{\frac{N}{A}\left(1+\frac{c_{2a}}{c_{1a}}\right)}$$

Substituting Eqs. (29) for C_1 and C_2 and Eqs. (9) and (10) for C_{1a} and C_{2a} the Error becomes

(30)
$$\text{Error} = m\,\frac{\frac{\sin\alpha}{\alpha}\left(1-\frac{m}{6}\right) - 2\cos\alpha + \frac{\sin\alpha}{\alpha}\cos\alpha\left(1+\frac{m}{6}\right)}{2(1-\cos\alpha)\left(\frac{\sin\alpha}{\alpha}\left(1+\frac{m^2}{12}\right) - \cos\alpha\left(1-\frac{m}{2}\right)\right)}, \quad \alpha \leq 90^\circ$$

$$\text{Error} = m\,\frac{\frac{\sin\alpha}{\alpha}\left(1-\frac{m}{6}\right)}{2\left(\frac{\sin\alpha}{\alpha}\left(1+\frac{m^2}{12}\right) - \cos\alpha\left(1+\frac{m}{2}\right)\right)}, \quad \alpha > 90^\circ$$

Equation (30) is plotted as a correction curve in Chart 2. Note that this correction is applicable only to the compressive stresses, because they are distributed over the thickness of the section. No correction for stresses in reinforcing steel is necessary since their distribution substantially satisfies the assumption made in the Introduction.


α, Degrees
180
170
160
150
140
130
120
110
100
90
1.00
.95
.90
.85
.80
.75
.80
.75
.70
.65
.60
.55
.50
E/R
E/R
CHART 1
RELATION OF α AND E = M/N
0
10
20
30
40
50
60
70
80
90
α, Degrees

683-12

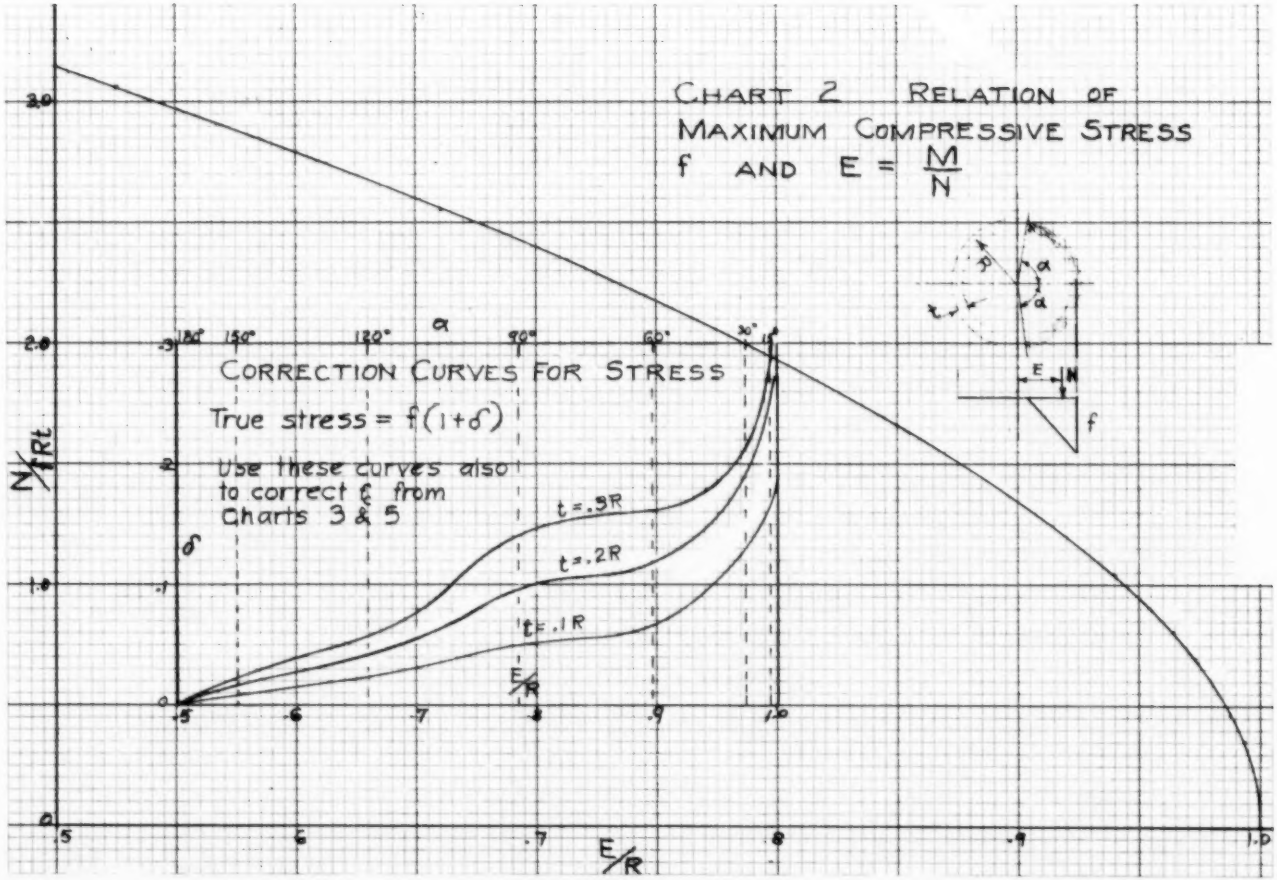


CHART 2 RELATION OF MAXIMUM COMPRESSIVE STRESS f AND E = M/N
CORRECTION CURVES FOR STRESS
True stress = f(1+δ)
Use these curves also to correct ε from Charts 3 & 5
N/πRt
δ
E/R
t=.3R
t=.2R
t=.1R
α
180°
150°
120°
90°
60°
30°
0
.5
.6
.7
.8
.9
1.0
1.5
2.0
2.5
3.0
E
N
f

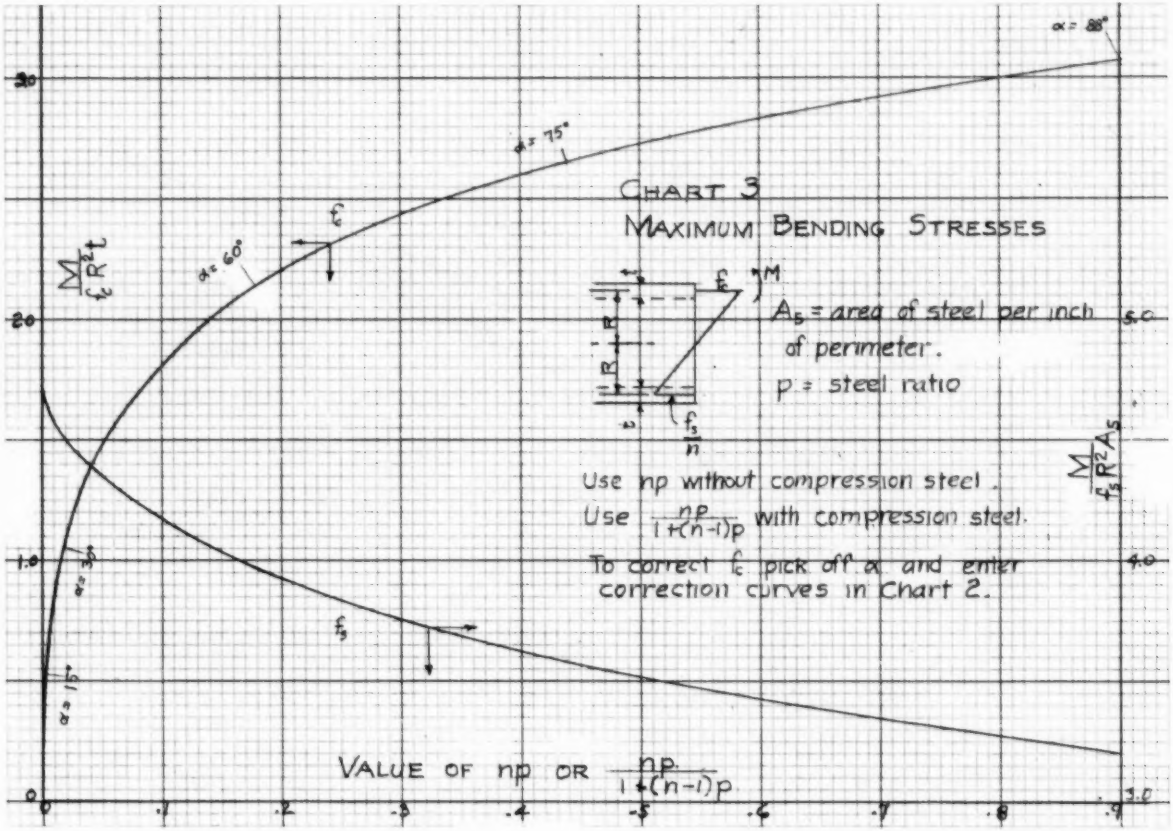


CHART 3
MAXIMUM BENDING STRESSES
A_s = area of steel per inch of perimeter.
p = steel ratio
Use np without compression steel.
Use $\frac{np}{1+(n-1)p}$ with compression steel.
To correct f_c pick off α and enter correction curves in Chart 2.
$\frac{M}{f_c R^2 t}$
$\frac{M}{f_s R^2 A_s}$
$\alpha = 15°$
$\alpha = 30°$
$\alpha = 60°$
$\alpha = 75°$
$\alpha = 88°$
f_c
f_s
VALUE OF np OR $\frac{np}{1+(n-1)p}$
0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
1.0
2.0
3.0
4.0
5.0

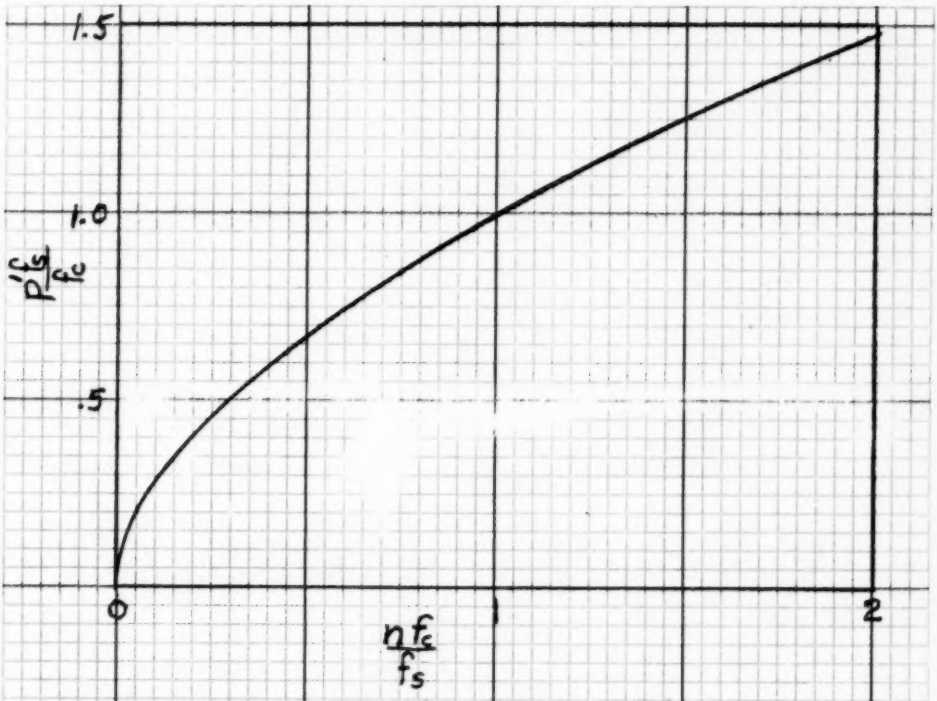


CHART 4. BALANCED STEEL RATIO

Without compression steel use $p = p'$ as obtained from Chart.

With compression steel use $p = \frac{p'}{1-(n-1)p'}$

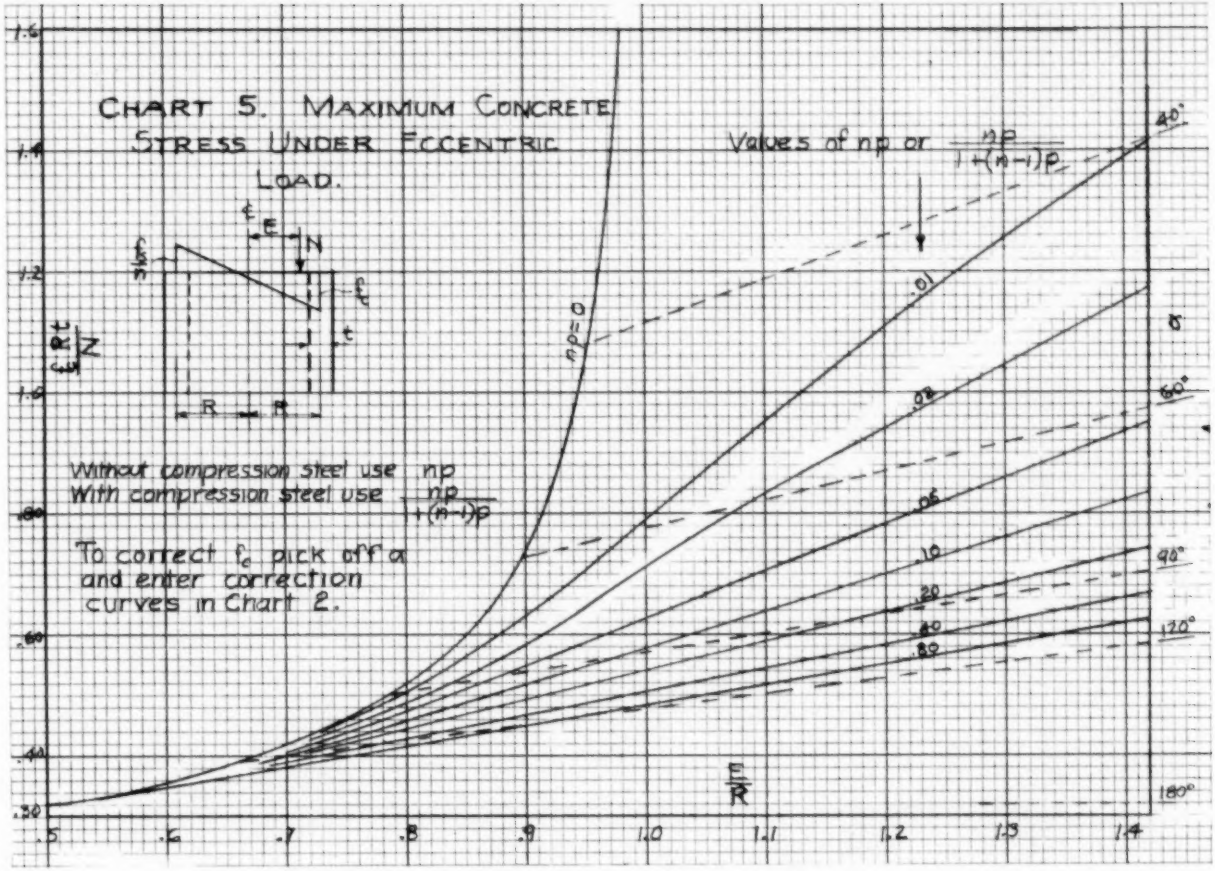


CHART 5. MAXIMUM CONCRETE STRESS UNDER ECCENTRIC LOAD.
Values of np or $\frac{np}{1+(n-1)p}$
Without compression steel use np
With compression steel use $\frac{np}{1+(n-1)p}$
To correct f_c pick off α and enter correction curves in Chart 2.
$\frac{f_c bt}{N}$
$\frac{e}{R}$
α
np=0
.01
.02
.05
.10
.20
.40
.80
40°
60°
90°
120°
180°
1.6
1.4
1.2
1.0
.80
.60
.40
.20
.5
.6
.7
.8
.9
1.0
1.1
1.2
1.3
1.4
E
N
R
t

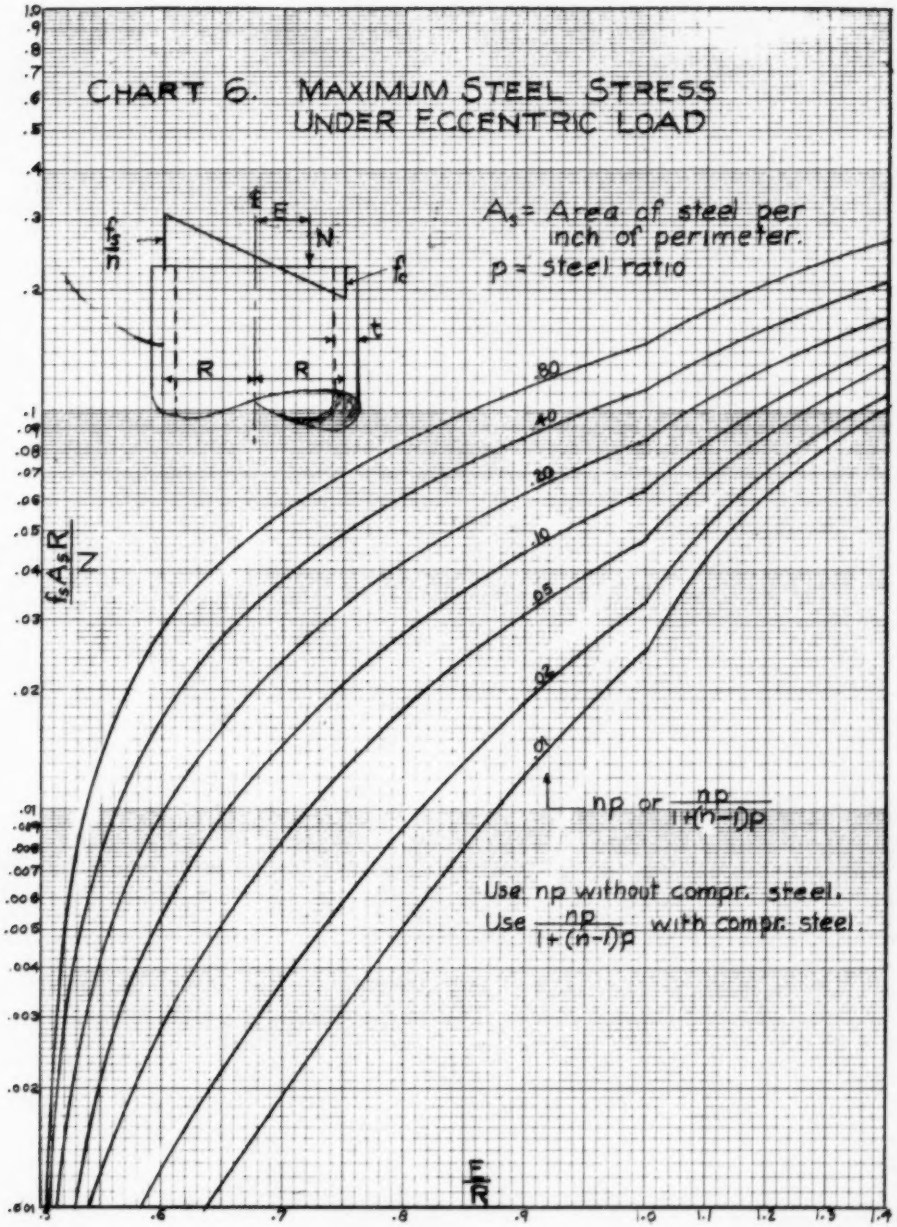


CHART 6. MAXIMUM STEEL STRESS UNDER ECCENTRIC LOAD
A_s = Area of steel per inch of perimeter.
p = steel ratio
E
N
R
R
t
f_c
$nf_c\frac{t}{E}$
$\frac{f_s A_s R}{N}$
$\frac{E}{R}$
.80
.40
.20
.10
.05
.02
.01
np or $\frac{np}{1+(n-1)p}$
Use np without compr. steel.
Use $\frac{np}{1+(n-1)p}$ with compr. steel.

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